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Journal of the

POWER DIVISION

Proceedings of the American Society of Civil Engineers

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October, 1956

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Journal of the
POWER DIVISION
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HYDRO-ELECTRIC POWER IN THE SOUTHEAST

C. P. Lindner,[1] and L. F. Johnson,[2] Members ASCE
(Proc. Paper 1086)

SYNOPSIS

The southeast has been endowed by nature with conditions favorable for the development of water power. Rainfall and runoff are high, stream profiles are relatively steep, and dam sites abound especially in the Mountain and Piedmont provinces. As a result water power was utilized at an early date. With the increase in use of electric energy, water-driven generating capacity increased with comparative rapidity. This continued until about 1930 when the construction of dams and hydro-electric plants by private utility companies stopped. Recently interest in further development of hydro-electric power has increased. The old cliché that practically all the best dam sites have been used has been found not to be true especially with respect to the construction of dams that create reservoirs for stream flow regulation and of dams downstream that will receive the benefit of this regulation. The Georgia Power Company completed one major hydro-electric project in the Altamaha River Basin in 1953, has increased its installed capacity on the Chattahoochee River, and has applied for a license to build a new dam and power plant on that stream. The Alabama Power Company is preparing plans for complete utilization of the head on the Coosa River in Alabama, has installed additional capacity in one of its plants on that stream, and has applied for a permit to investigate the feasibility of constructing one dam and two hydro-electric plants in the Warrior River Basin in Alabama.

The Warrior River Electric Cooperative Association proposes to build three dams and power plants also in the Warrior Basin. The Carolina

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Aluminum Company is planning a new dam on the Yadkin River. Part of the impetus was furnished by Federally constructed and authorized projects, some of which firmed the stream flow and made development more desirable from an economical standpoint. At the present time (end of 1955) there are 8 Federally authorized multiple purpose projects including power in the southeast. Two have been completed and 4 are under construction. With a large potential and a reawakened interest, the future of the development of hydroelectric projects and their use in the southeast continues to appear bright.

The Southeast

The "Southeast" as referred to in this paper is restricted to the region within the Continental limits of the South Atlantic Division of the Corps of Engineers, U. S. Army. This area includes the river basins draining to the Atlantic Ocean and the Gulf of Mexico between northern North Carolina and the Mississippi River Basin in Mississippi. Nearly all of North Carolina, South Carolina, Georgia, and Alabama: all of Florida; and eastern Mississippi lie within this region. Power supply areas 21, 22, 23 and 24 of the Federal Power Commission correspond quite closely to it.

Physical Features Affecting Hydro Power Development

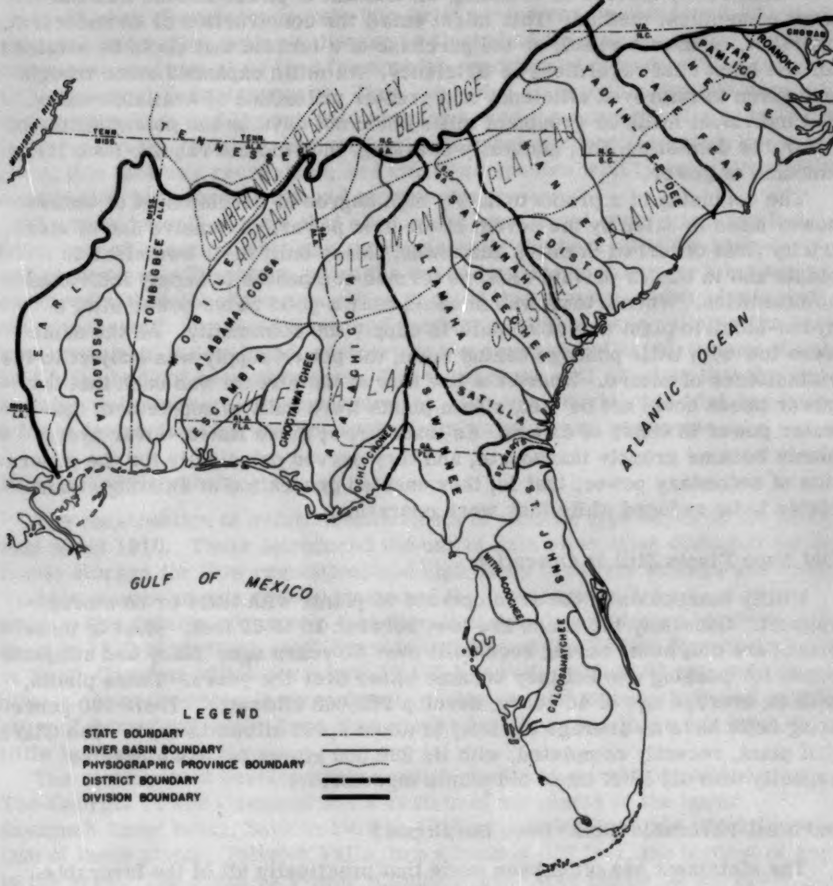
Figure 1 shows the river basins of the area. Several of the basins extend into the Blue Ridge mountain and Piedmont Plateau provinces. These provinces are the major hydro areas. There is only one hydro plant of large size in the coastal lowlands beyond, although two are now (1955) under construction. The best power basins are the five which extend into the mountains—the Yadkin-Pee-Dee, Santee, Savannah, Apalachicola, and Alabama-Coosa; about 97 percent of the developed hydro power of the region is concentrated there. The headwater streams of those basins are fed by rain in the high altitudes where the annual precipitation averages as much as 84 inches. When the streams descend to the Piedmont Plateau, where most of the dam sites are located, their flow is well established. Flows in the upper portions of the five basins average 2 cubic feet per second, or more, per square mile. The flows of streams originating in the Piedmont Plateau average much less—between 1.5 and 1 cubic foot per second per square mile.

The border between the Piedmont and the Coastal Plain, or Fall Line, is a zone of rapid fall and well established flow. Many of the first hydro plants and several of the major later developments were constructed in that zone.

History of Hydro Power Development

Utilization of water power in the Southeast has closely paralleled historical development in the remaining eastern seaboard states. Earliest use was made in grist mills and saw mills followed by the employment of water driven equipment in textile mills. Most early industries requiring mechanical power were located near falls or steeply sloping streams where sufficient power for the purpose could be developed at low cost. Cities grew near such sites and received their first impetus from the need for power.

These first mechanical power installations were crude, for the design of



RIVER BASINS
OF
SOUTH ATLANTIC DIVISION

SCALE IN MILES
50 0 50 100

Figure 1.

the water wheel had not been perfected and the science of turbine settings was little understood. Only a limited amount of power was required and it was generally available in over-abundance so that efficiency was not a factor to be considered nor was it desirable to develop the potentialities of a site. The least costly method of obtaining the amount of power needed was the most economical method. This might entail the construction of an undershot, overshot, or breast wheel, or the purchase of a turbine that could be obtained for the least cost regardless of efficiency. As mills expanded more thought was given to improved efficiency and greater utilization of available water. But individual mills or groups of mills could not develop the potentialities of favorable dam sites, for, generally speaking, they did not require such large amounts of power.

The beginning of a proportionately high degree of development of water power sites is dated by the advent of electric power. Extensive use of electricity first occurred in cities and towns. Many built their own electric plants and in others private citizens formed companies to supply individual communities. When a town was situated near a good water power site, a hydro-electric plant was often built to supply the community. As the dams were low with little pondage behind them, the power supply was subject to the vicissitudes of nature. Where the low flow of the stream was such that the power needs could not be met, steam plants were built to supplement the water power in times of drouth. As loads grew, these run-of-river hydroplants became grossly inadequate, and they served principally for the generation of secondary power; that is, they enabled generation at existing steam plants to be reduced while they were operating.

Old Type Plants Still in Operation

Utility companies in the area operate 55 plants with little or no storage support. Generally the heads are low, between 10 to 60 feet. Most of these plants are old, some having been built over 50 years ago. Many had adequate ponds for peaking use but they became silted over the years. These plants, with an average age of 40 years, develop 250,000 kilowatts. Their 180 generating units have an average capacity of about 1,400 kilowatts each. The Clark Hill plant, recently completed, with its 280,000 kilowatts, has a greater capacity than all 55 of these old plants together.

Have All Favorable Sites Been Developed?

The statement has often been made that practically all of the favorable hydro-electric sites have been utilized. When this statement is made, what is being thought of is the old type of plant, with its minimum dam and use of a local falls or shoals. It was true only for the conditions that existed early in the 20th century. As transmission systems grew, and vast areas were interconnected, and as the financial capabilities of utility companies enlarged, the conception of the method of utilization of hydro-electric power changed. Because of diversity in timing between low flows on various streams and between low flow and the peak load, it was found that dependable capacity could be attributed to some of the run-of-stream plants. Moreover, storage reservoirs could regulate the stream flow, and pondage available permitted the utilization of the entire regulated flow in moderate and low flow periods at high rates during a few hours each week or each day. Since most of the storage plants are moderate or high head plants additional capacity can be

installed to utilize the flows at high rates generally at less cost per kilowatt than the cost of steam capacity, and this hydro capacity is much less expensive to maintain and operate than the corresponding amount of steam capacity. Further, even large hydro-electric units can be started, brought to speed, synchronized, connected to the line, and brought to full load in about 8 minutes. In case of emergency even this brief period can be shortened. Also, when not generating, hydro units can be floated on the line for power factor correction, and run at no load from which condition of operation they can pick up load almost instantaneously. Thus the hydro plant with a storage reservoir is ideally suited to peak load operation, and the economy of a hydro plant is increased by its design and utilization for that purpose. As peak loads grew, this fact was recognized, and utility companies with financial capability constructed peak load plants and storage reservoirs.

It should be self-evident that sites which were most economical for low head, run-of-stream plants are not necessarily the best sites for high dams and reservoirs. Therefore, when the water resources of our streams were surveyed with the modern conception in mind that storage-power plants and other plans with regulated flows and pondage available were suitable for peak load operation, it was found, contrary to the common belief, that many of the best sites had not been developed. Yet the thought still persists that the remaining water powers are uneconomical. Beyond a doubt, this idea has been a large factor delaying development of our water power resources.

Modern Developments

The construction of hydro developments of modern type began in the southeast about 1910. These introduced the use of dam sites often distant from the loads; storage for flow regulation; and high dams to create storage and head. Turbine improvements greatly increased the power and efficiency of hydro units. One notable advance in that direction was demonstrated by the installation in 1917 of vertical-shaft Francis turbines at the Yadkin Narrows plant in North Carolina which developed 17,100 kilowatts each at 91 percent efficiency. Improvements in size, details of design, and manufacture and installation of Francis turbines have been made since then, but there has been very little increase in efficiency.

The more recent developments consist mostly of groups of related plants. The Georgia Power Company has a system of six plants in the upper Savannah River basin, built in 1910 to 1927, which develops 166,000 kilowatts. One of these plants, Tallulah Falls, has a head of 600 feet, the highest of any in the area. The Alabama Power Company has three plants on the lower Coosa River built in 1910 to 1921 which develop 237,000 kilowatts and produce over 1 billion kilowatt-hours a year from a fall of 225 feet. Another group of three on the Tallapoosa, built by the same company in 1922 to 1930, develops 236,000 kilowatts with a fall of 280 feet. Martin Dam, the upstream one of that group, has a power storage of 1,375,000 acre feet—one of the largest amounts in the region. Twelve plants of the Duke Power Company completed by 1928, which develop 444,000 kilowatts out of a fall of 770 feet on the Wateree-Catawba River in the Santee basin, make more complete use of the potential of a large river than any group in the area.

On the Yadkin River, three Carolina Aluminum Company plants develop 135,000 kilowatts. The Bartletts Ferry and Goat Rock plants of the Georgia Power Company generage 91,000 kilowatts on the Chattahoochee River.

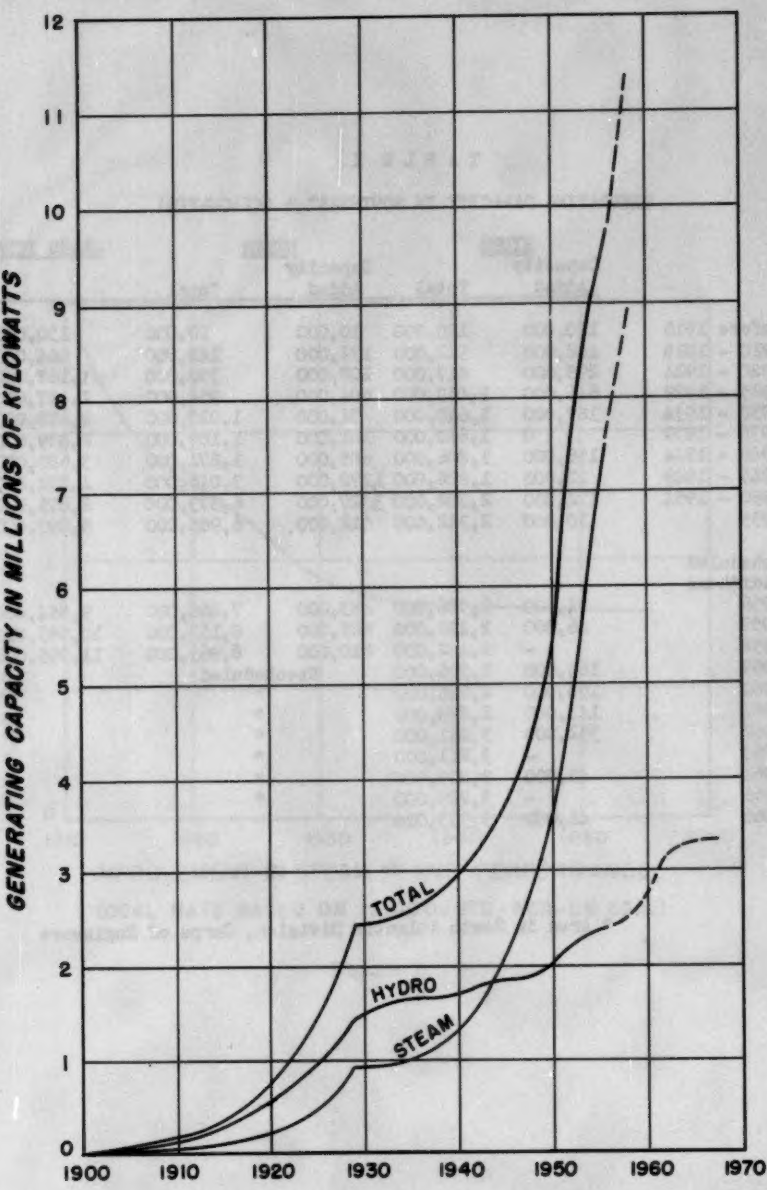
In addition to the above systems there are several individual plants with storage reservoirs and considerable power capacity. The Saluda plant of the South Carolina Electric and Gas Company, completed in 1930, generates 130,000 kilowatts on the lower Saluda River in the Santee basin and has a storage of 1,614,000 acre feet. The Santee-Cooper development in the lower Santee basin, completed in 1942 by the South Carolina Public Service Authority, diverts the flow of the Santee to near tide level in the adjoining Cooper River basin, developing about 135,000 kilowatts with 70 feet of head; it uses a storage of 1,560,000 acre feet. The Allatoona project, completed by the Corps of Engineers in 1950, and operated by that agency, generates 74,000 kilowatts on the Etowah River in the Alabama-Coosa basin and has a power storage of 229,000 acre feet. The Clark Hill project, also constructed and operated by the Corps of Engineers, was completed in 1953. The installed capacity is 280,000 kilowatts; the rated head is 136 feet, and the usable power storage is 1,340,000 acre feet. The Tillery plant on the Pee-Dee River (62,000 kilowatts), the Buzzards Roost plant on the Saluda (15,000 kilowatts) and the Lloyd Shoals and Sinclair plants in the Altamaha basin (14,000 and 45,000 kilowatts respectively) are the other major hydro plants of the area.

The above 37 major plants develop 2,064,000 kilowatts. They have 141 generating units which average 14,600 kilowatts each, or about ten times the power of the units of the older type of plant. The "old" and "modern" hydro plants together are 92 in number, have 321 generating units, and develop 2,312,000 kilowatts. The total reservoir storage capacity in all hydro plants of the area is 8,200,000 acre feet.

Growth of Electric Energy Producing Capacity

The growth of hydro capacity continued at a steadily increasing rate from the early days up to 1930. Until that time practically all hydro construction was by utility companies. Since then only one hydro plant, Sinclair, in the Altamaha Basin, has been completed by those companies, but some hydro capacity has been installed at their existing plants. From 1930 to 1940, when the Buzzards Roost plant was completed by Greenwood County, South Carolina, no hydro plants were built. In 1942 the Santee-Cooper development was completed, in 1950 and Allatoona project began operation, in 1953 and 1954 the seven units at Clark Hill project were put on the line, and in 1953 Sinclair Dam was completed. Figure 2 and Table 1 illustrate the growth of both hydro and steam capacity.

Steam power construction began slowly, and by 1920 the ratio was nearly four to one in favor of hydro capacity. Revolutionary improvements in efficiency of steam plants, (See Figure 3) reduction in the cost of construction, and increases in size of individual units were being made which greatly improved the economy of steam generation. A severe drought in 1925 reduced hydro generation considerably that year and forced a downward revision of the estimated dependable capacity of the hydro systems of that time. For these reasons, and because of the smaller investment and shorter construction time required, and hence less risk if the loads failed to develop as expected, the utility companies changed to steam construction late in the Twenties. By 1943 steam capacity equalled hydro; in 1952 it was double hydro in amount; and at the present time (1955) it is triple hydro.



GENERATING CAPACITY IN THE SOUTHEAST

Figure 2.

TABLE 1

GENERATING CAPACITY IN SOUTHEAST * (KILOWATTS)

	HYDRO		STEAM		GRAND TOTAL
	Capacity Added	Total	Capacity Added	Total	
Before 1910	120,000	120,000	10,000	10,000	130,000
1910 - 1919	402,000	522,000	132,000	142,000	664,000
1920 - 1924	295,000	817,000	208,000	350,000	1,167,000
1925 - 1929	646,000	1,463,000	604,000	954,000	2,417,000
1930 - 1934	187,000	1,650,000	74,000	1,028,000	2,678,000
1935 - 1939	0	1,650,000	161,000	1,189,000	2,839,000
1940 - 1944	156,000	1,806,000	685,000	1,874,000	3,680,000
1945 - 1949	22,000	1,828,000	1,172,000	3,046,000	4,874,000
1950 - 1954	474,000	2,302,000	3,327,000	6,373,000	8,675,000
1955	10,000	2,312,000	612,000	6,985,000	9,297,000
Scheduled Additions					
1956	74,000	2,386,000	483,000	7,468,000	9,854,000
1957	46,000	2,432,000	685,000	8,153,000	10,585,000
1958	-	2,432,000	810,000	8,963,000	11,395,000
1959	164,000	2,596,000	Unscheduled		
1960	150,000	2,746,000	"		
1961	143,000	2,889,000	"		
1962	352,000	3,241,000	"		
1963	-	3,241,000	"		
1964	48,000	3,289,000	"		
1965	-	3,289,000	"		
1966	44,000	3,333,000			

* Area in South Atlantic Division, Corps of Engineers

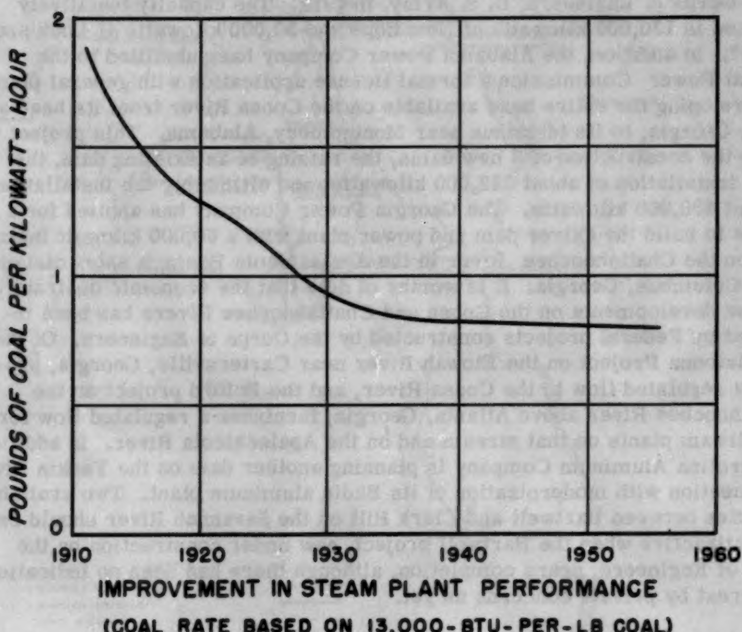

POUNDS OF COAL PER KILOWATT - HOUR
2
1
0
1910
1920
1930
1940
1950
1960
IMPROVEMENT IN STEAM PLANT PERFORMANCE
(COAL RATE BASED ON 13,000-BTU-PER-LB COAL)

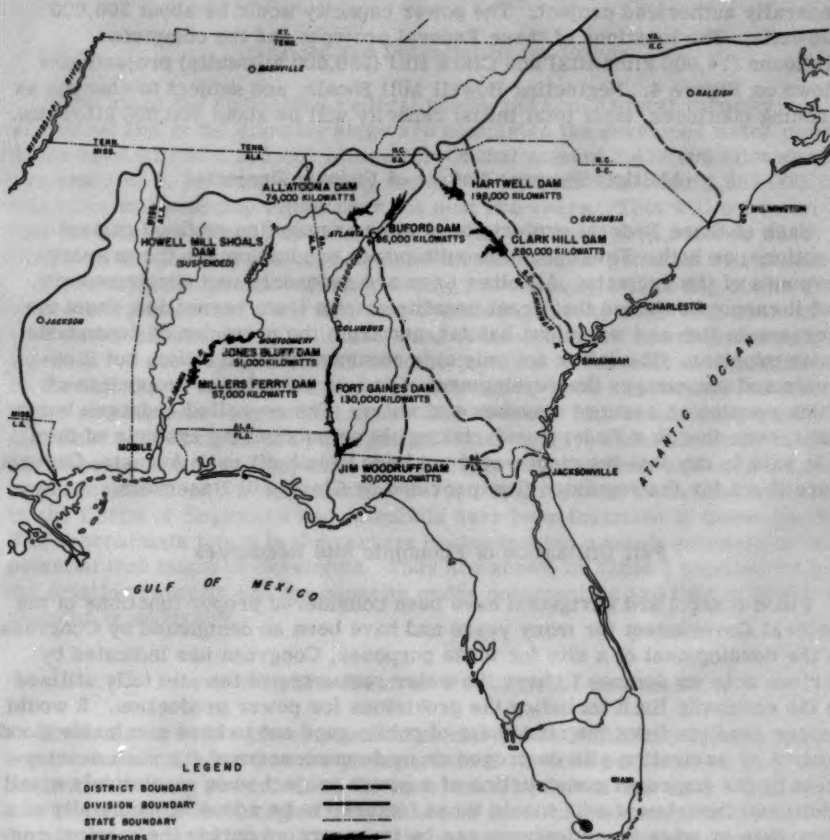
Figure 3.

Increased Interest by Private Enterprize, in Hydro-Electric Development

In recent years, the interest of private utility companies in hydro-electric development has been renewed, and cooperatives have become interested in such development as well. The Warrior River Electric Cooperative Association is preparing to construct two plants on the Locust Fork of the Black Warrior, a stream located in the Tombigbee basin in Alabama. These plants are expected to have a capacity of 70,000 kilowatts. The Alabama Power Company has applied to the Federal Power Commission for a preliminary permit to build a dam and power plant at the Blevins Hollow site on Sipsey Fork of the Black Warrior River and to install hydro-electric capacity at Lock and Dam 17 on the Black Warrior River, a navigation project completed by the Corps of Engineers, U. S. Army, in 1917. The capacity tentatively proposed is 130,000 kilowatts at New Hope and 30,000 kilowatts at Lock and Dam 17. In addition, the Alabama Power Company has submitted to the Federal Power Commission a formal license application with general plans for developing the entire head available on the Coosa River from its head at Rome, Georgia, to its terminus near Montgomery, Alabama. This project involves the construction of 4 new dams, the raising of an existing dam, the initial installation of about 283,000 kilowatts, and ultimately the installation of about 400,000 kilowatts. The Georgia Power Company has applied for a license to build the Oliver dam and power plant with a 60,000 kilowatt installation on the Chattahoochee River in the Apalachicola Basin, a short distance above Columbus, Georgia. It is worthy of note that the economic desirability of these developments on the Coosa and Chattahoochee Rivers has been increased by Federal projects constructed by the Corps of Engineers. Of these, the Allatoona Project on the Etowah River near Cartersville, Georgia, provides a regulated flow to the Coosa River, and the Buford project on the Chattahoochee River above Atlanta, Georgia, furnishes a regulated flow for downstream plants on that stream and on the Apalachicola River. In addition, the Carolina Aluminum Company is planning another dam on the Yadkin River in connection with modernization of its Badin aluminum plant. Two available dam sites between Hartwell and Clark Hill on the Savannah River should become attractive when the Hartwell project, now under construction by the Corps of Engineers, nears completion, although there has been no indication of interest by private concerns as yet.

Federal Hydro Capacity in the Southeast

The Federal Government has under construction by the Corps of Engineers, 4 multiple purpose projects with power installations. Jim Woodruff Lock and Dam on the Apalachicola River with a power capacity of 30,000 kilowatts will be completed in 1956. Buford Reservoir on the Chattahoochee River will be completed in 1957. The hydro installation there will total 86,000 kilowatts. Hartwell project on the Savannah River to be completed about 1962 is planned for an initial installation of 198,000 kilowatts and an ultimate capacity of 330,000 kilowatts. Ft. Gaines Lock and Dam on the Chattahoochee River with an installed capacity of 130,000 kilowatts is also scheduled for completion about 1962. Two other projects have been authorized by Congress that when constructed will contain power producing facilities. These are the Jones Bluff and Millers Ferry Lock and Dams on the Alabama River. The tentatively considered capacities are, respectively, 51,000 and 57,000 kilowatts. Upstream, on the Coosa River, the Howell Mill Shoals project was authorized



CORPS OF ENGINEERS AUTHORIZED PROJECTS WITH POWER

SCALE IN MILES

Figure 4.

for Federal development until 1954 when Congress suspended the authorization to permit development of the Coosa by the Alabama Power Company in the manner mentioned above. Should the Alabama Power Company decide not to proceed with its plan the Howell Mill Shoals project will again become a Federally authorized project. The power capacity would be about 200,000 kilowatts. The locations of these Federal projects and the completed Allatoona (74,000 kilowatts) and Clark Hill (280,000 kilowatts) projects are shown on Figure 4. Neglecting Howell Mill Shoals, and subject to changes as planning continues, their total initial capacity will be about 906,000 kilowatts.

Multiple Purpose Nature of Federal Projects

Each of these Federal projects have either navigation or flood control functions, or both. These together with power production are the primary purposes of the projects. All other uses are secondary and supplementary. But it cannot be denied that great benefits accrue from recreation, from the increase in fish and waterfowl habitat, and from the provision of dependable water supplies. The latter not only aids communities and cities, but it attracts and encourages the development of industries a large proportion of which require an assured abundance of water. The so-called hydrogen bomb plant, even though a Federal undertaking, is an outstanding example of this. It is safe to say that the plant would not have been built near Augusta, Georgia, were it not for the regulated flow provided by Clark Hill Reservoir.

Full Utilization of Economic Site Resources

Flood control and navigation have been considered proper functions of the Federal Government for many years and have been so designated by Congress. In the development of a site for these purposes, Congress has indicated by various acts its desires to have the water resources of the site fully utilized to the economic limit including the provisions for power production. It would appear sensible from the standpoint of public good not to have a valuable flood control or navigation site destroyed or made uneconomical for such development by the improper construction of a power project when a relatively small additional investment will enable those features to be added economically at a later date or when those features can be incorporated during the original construction for an added cost that is less than the benefits to be gained. Likewise a valuable power site should not, in effect, be destroyed by the construction of a flood control or navigation project in such a manner that will prevent the economical development of power at the site.

Necessity for Multiple Purpose Development in the Southeast

In the southeast, because of the relatively narrow flood plains, the construction of dams and reservoirs generally cannot be justified by the benefits from flood control alone or from flood control and navigation combined. But because of the many good dam and reservoir sites and the high average runoff, power development is often worthwhile; and the navigation and flood control features can be justified economically when added to the power development. Except for an occasional small local protection project and levee and

major drainage projects in the coastal lowlands most flood control projects, and generally the more difficult navigation projects could not be built in the southeast because of lack of economic justification were it not for the fact that power development is capable of sustaining a major portion of the cost.

Developed and Potential Hydro Power

When all of the authorized Federal plants and non-Federal capacity scheduled and in the planning stage are completed the developed water power of the area will be 3,521,000 kilowatts. Of that amount 2,432,000 kilowatts are expected to be in operation by the end of 1958 and the other 1,085,000 kilowatts will probably follow over the next few years. This will not mean that the ultimate development of water power will have been reached when the above program is accomplished, as a considerable potential will remain.

The utility companies will undoubtedly add capacity at existing plants where there is room. They may find it advantageous to redevelop some of their older plants, and to build new ones at sites that are now economically justified for development or will become justified as a result of regulation to be furnished by reservoirs under construction or to be constructed. Federal hydro development is expected to continue where it may be advantageously combined with flood control, navigation, water supply, and other general improvements. Many hydro plants in addition to those already authorized have been recommended for construction in reports submitted from time to time by the Corps of Engineers and potentials have been indicated in these reports. The approximate totals in the various basins furnish a rough estimate of the potential that might be developed. They are shown in Table 2 together with the existing capacity and the capacity under construction and that on which planning is underway.

Future of Hydro-Electric Power in the Southeast

Much of this hydro power potential of more than 5 million kilowatts can be most economically used in the peak portion of the load curve. This is a large amount of peaking power. Lest the conclusion be drawn that such a quantity may never be utilized, cognizance should be taken of the growth of the load in the southeast. In 1920 or 1930 only the most intrepid prognosticators would have forecast the load that now exists. Loads have grown and are continuing to grow almost by geometrical progression. Electrical World, 21 January 1952, page 7, showed that the margin between peak load and installed capacity in 1951 in the southeast area, a somewhat larger area than the one defined herein, was only about 2.4%, the lowest margin for any region in the country. Although this situation has been remedied so that a satisfactory margin now exists, Federal Power Commission estimates indicate that the southeast regional reserve capacity will again, in the near future, fall below the desirable 15% reserve and that, unless construction is accelerated, the deficiency may be serious in a few years. The electric companies and generating unit manufacturing concerns were, for a time, hard pressed to keep construction ahead of the demand and this condition could be repeated with continued rapid growth of demand coupled with the diversion of electrical manufacturing effort and facilities to the production of other urgently needed electrical goods and equipment. Defense loads grew rapidly in the recent past and are

TABLE 2

HYDRO POWER SUMMARY
(Kilowatts)

River basin	Existing	Under construction & definitely planned	Potential	Total
Tar	-	-	-	-
Neuse	1,500	-	-	1,500
Cape Fear	5,000	-	15,000	20,000
Yadkin-Pee Dee	224,500	40,000	290,000	554,500
Santee	823,000	-	410,000	1,233,000
Edisto	-	-	-	-
Savannah	474,000	198,000	380,000	1,052,000
Ogeechee	-	-	-	-
Altamaha	68,000	-	64,000	132,000
Satilla	-	-	-	-
St. Marys	-	-	-	-
St. Johns	500	-	-	500
Withlacoochee	3,200	-	-	3,200
Suwannee	-	-	-	-
Ochlockonee	8,800	-	-	8,800
Apalachicola	139,000	310,000	290,000	739,000
Choctawhatchee	1,400	-	-	1,400
Escambia	7,600	-	-	7,600
Mobile(Alabama-Coosa)	555,500	391,000	400,000	1,346,500
Mobile (Tombigbee)	-	270,000	80,000	350,000
	2,312,000	1,209,000	1,929,000	5,450,000

continuing to grow, and there has been no cessation or levelling off of civil demand. This is an electrical age, and no end can be seen to the increasing use of electricity. The time may come in the not far distant future when all the economical potential hydro capacity in the southeast will have been developed. When that happens the end of further hydro-electric development in the area will have been substantially reached, and it will be necessary to seek additional capacity through other sources of power only. But the operating plants will continue to do their chore of serving the nation by conserving natural resources. This is a valuable service which has not been evaluated in establishing the economic justification of hydro-electric projects, for when other resources are developed and used they are depleted; in contrast, when hydro power is used, the resource is not depleted, for its supply is continually replenished and remains undiminished, subject only to the vicissitudes of nature many of the effects of which are eliminated by the regulation provided by reservoir projects.

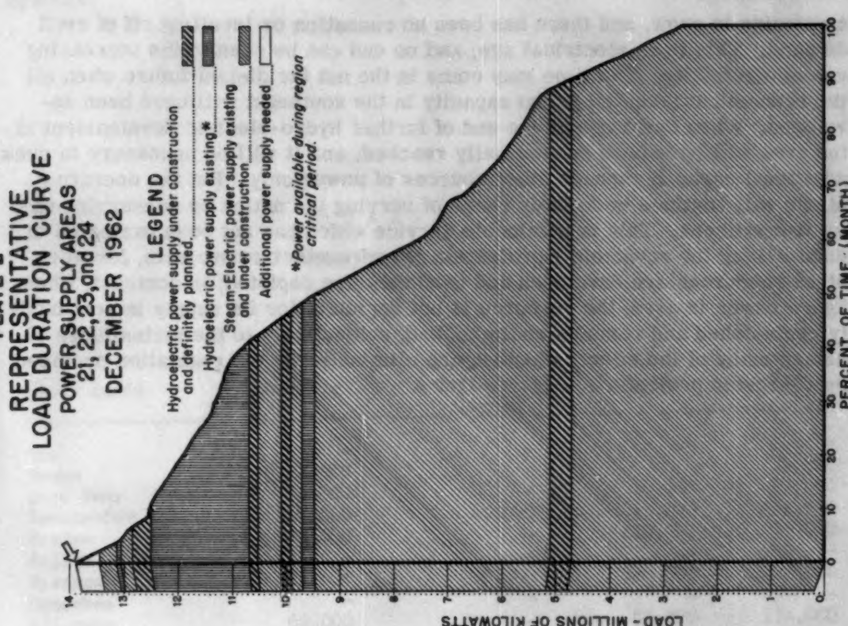


PLATE 2
REPRESENTATIVE LOAD DURATION CURVE
POWER SUPPLY AREAS 21, 22, 23, and 24
DECEMBER 1962

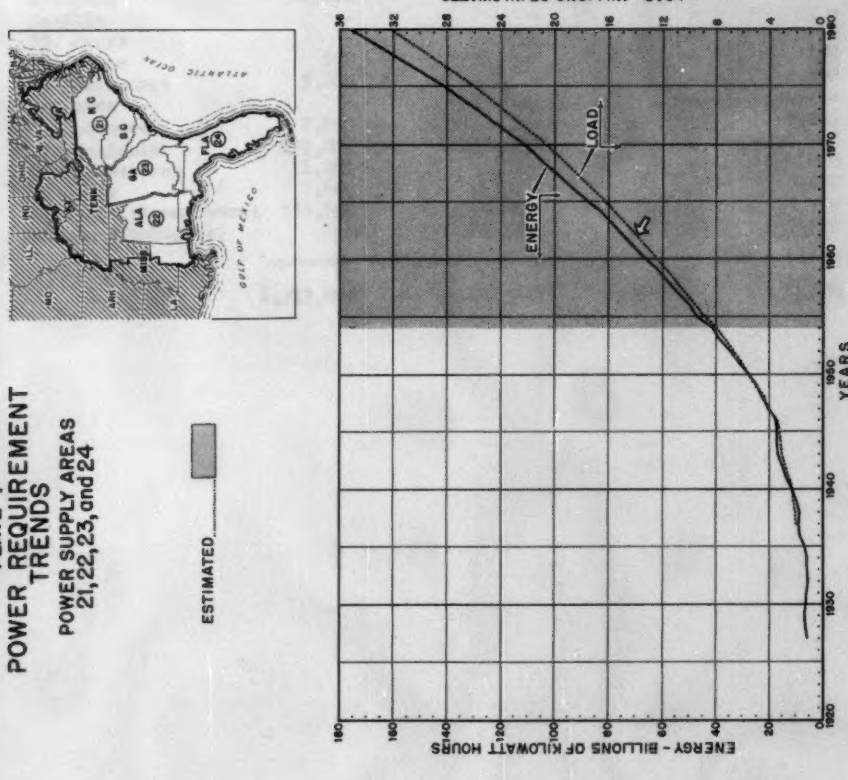


PLATE 1
POWER REQUIREMENT TRENDS
POWER SUPPLY AREAS 21, 22, 23, and 24

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

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DISCUSSION
(Proc. Paper 1094)

Journal of the

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DISCUSSION

(Proc. Paper [illegible])

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Discussion of
"MODEL TESTS, ANALYTICAL COMPUTATION AND OBSERVATION OF AN ARCH DAM"

by M. Rocha, J. Laginha Serafim, A. F. da Silveira, and
J. M. Ressurreição Neto
(Proc. Paper 696)

M. ROCHA,* J. LAGINHA SERAFIM,** MEMBERS ASCE, A. F. DA SILVEIRA,*** and J. M. RESSURREIÇÃO NETO.***—The writers are indebted to the discussers for the attention given to their paper. The questions raised by some of the discussers call for further elucidation of certain points which, due to limited space, were not perhaps fully dealt with. The need for condensing the paper led the writers to omit many aspects and results obtained in the investigations carried out.

The writers know from their own experience on the "trial load" method what the effect of tangential displacements and rotations is on the reduction of the stresses obtained by radial adjustments. The results presented by Mr. Copen in relation to Yellowtail Dam agree fully with what is to be expected from this effect: when the dam supports loads by tangential stresses it undergoes less radial displacements and therefore the normal stresses in the arches and cantilevers are lower. It is to be noted however that, considering only the effect of hydrostatic pressure, the reductions of the maximum normal stresses through the introduction of the tangential shear and twist do not, as a rule, go beyond 30% either for the maximum tensile stresses or for the maximum compression stresses. This, which is well known from various Technical Memoranda of the Bureau of Reclamation, has also been checked from comparisons of results of model studies of Portuguese dams with their "trial load" calculations.

In the case of the study in question the writers made altogether three simplified "trial load" analyses, the first one with uncracked cantilevers.

Even though this calculation showed that the dam would crack at the heel, the calculation presented was only made after the observation of the prototype from which it was decided what depth of crack should be considered.

Bearing in mind what has been said above and also the results of these calculations, the writers were able to conclude that the stresses determined by a complete calculation could never, for the cracked zones of the dam under study, give values of the order of those given by the models or observed in the prototype once they differed much more than 30% (see zones at points 1u and 2d). In the writers' opinion the "trial load" calculation cannot, in view of the fact that it is based on formulae of the Strength of Materials, be adequately applied to singular situations which occur, for example, near a crack or a

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hole. This is one of the reasons why the writers considered it unnecessary to proceed in this case with a complete "trial load" calculation which would undoubtedly cost a lot of money and time as cracked cantilevers had to be considered. In fact the writers, in an application of the complete "trial load method" for another arch dam, took about four times as long with the complete calculation as they did with a calculation with only radial adjustments. At the end they did not consider that the information obtained compensated for the difference in time.

Mr. Copen finds that "The greatest value derived from the "trial load method is not necessarily the final numerical values of stresses and displacements, but rather the determination by comparative studies of the most economical structure." This is also an advantage that the writers have found in models, as various studies can be made on the same model very economically, such as knowing the influence of a crack or an opening, or studying a thinner dam simply by removing some of the material from the model. It is to be noted too that a model also makes it possible to obtain quite easily the stresses for all water levels of the reservoir and this is not so easy with the calculations.

The writers would also like to mention that whilst the analytical calculation of a dam becomes extremely laborious when the arches are not circular and the dam has a double curvature, as is the case of very recent rational designs for cupola dams, model studies are no more difficult in this case than in the case of a dam having a circular cylindrical face and uniform thickness. The cases of dams having four centered circular arches or having parabolic or eliptical arches which have been or are being studied in Portugal well demonstrated how the "trial load" calculation becomes more complicated for these cases.

It should be mentioned here that the dam studied (the Castelo do Bode cofferdam) had arches of constant thickness and consequently there was no simplification in the calculation of these elements.

When referring to the probable cost of model tests carried out in the L.N.E.C., Mr. Copen implies that the tests on the coffer dam lasted without interruption from 1947 to 1951 and further implies that the sentence "a later detailed report will cover the whole series of model tests carried out during the last 8 years" refers to the model study of only one dam. However, neither were the tests continued without a break nor does the eigth years' experience apply to one dam rather to various series of studies made on various dams.[1]

When considering the comparative costs of the complete "trial load" analysis and those of model studies the facilities, the equipment and the tradition which form the background of the question have to be considered. The establishment of a laboratory such as that which exists in Portugal costs money as does the establishment of an organization like the Bureau of Reclamation. Once the organizations exist it may be cheaper to do model tests than computations in one country or vice-versa. The only question which still stands is that of the scope of either of the methods in resolving the given problems and the writers believe that the one of model tests is greater.

With regard to the statement that when there is asymmetry, irregular

1. See Rocha, M., J. Laginha Serafim and A. F. da Silveira—"Design an Observation of Arch Dams in Portugal"—Paper presented to the Symposium on Arch Dams A.S.C.E. Convention, Knoxville 1956.

lines of insertion, heterogeneous foundations, irregularities in shapes, cracks, artificial abutments, spillway openings the "trial load" method becomes very laborious and expensive, Mr. Copen says that objections to heterogeneous foundations may largely be eliminated by special preparation of the foundation area and shape singularities such as cracks may be easily avoided by careful design. The writers are, however, of the opinion that, in spite of the suitable techniques which are available today for foundation treatment, such heterogeneity cannot be avoided and also as cracks sometimes occur, it is important to have methods for the study of their effects.

It is true that the model tests only consider the effect of hydrostatic pressure. The comparison of the results of observation of the structures with the results of model tests can be made once studies are made to separate the effect of different loadings on the structure. Such studies are possible and are being undertaken by the L.N.E.C. On the other hand, comparisons made between model results and observations of the structures in which the different effects were separated gave truly remarkable agreement,[1] which proves that the disadvantage of the other loadings not being considered in the models is not very great. Besides this it should not be forgotten that it is the hydrostatic pressure that stresses the whole mass of the concrete although the temperature can, in zones covering relatively small volumes, produce high stresses (very much higher than those considered in the classical "trial load"). But this point will be treated later.

Finally when referring to the calculation methods for arch dams attention should be drawn to the remarkable British contribution for the resolution of the problem by relaxation methods which is based directly on the Theory of Elasticity.[2]

Mr. Simonds called special attention to the important model studies of the Hoover Dam which undoubtedly were a notable step in the progress in knowledge of arch dams. His reminder is very timely even though in that case, due to lack of adequate equipment, reliable stress measurements on the upstream face were not made. Such tests showed that the complete "trial load" method is a suitable calculation method for Hoover type dams (arch-gravity), but it cannot be assumed that such a conclusion is also valid for thin arch dams. The tests carried out by the L.N.E.C. on Cabril, Salamonde, Caniçada and Bouçã dams, all thin arch dams with vertical curvature, showed that a complete "trial load" method for the calculation of these dams could not omit vertical displacements, which in these cases are very important, and which the "trial load" method as developed by the Bureau of Reclamation does not consider.

Mr. Richardson makes some very interesting remarks about the influence of certain factors, such as temperature, on the behaviour of the dams. Recently it has also been possible to gather some very enlightening results on this problem from observation of structures in Portugal.[1] The reasons pointed out for the differences which are found between the calculations, models and observations should be thoughtfully considered. It is to be hoped that, with the notable studies which are in progress in various countries on

1. See footnote on page 1094-4.
2. Allen D. N., L. Chitty, A.J.S.P. Pippard and R. T. Severn—The Experimental and Mathematical Analysis of Arch Dams, with special Reference to Dokan—Paper no. 6113 presented for discussion on Jan. 1956 at the Institution of Civil Engineers.

measurements in the actual structures, it will soon be possible to make a useful revision of the design criteria of dams.

A result of the lack of space led to the criticism of a sentence which did not quite express the writers' thought. Of course the writers could not forget that the large and important dams of the Bureau of Reclamation and of the T.V.A. were the object of careful and detailed calculations during their design. The writers wished to say that, as far as they know "that in all of those investigations no attempt was made to compare the observed stresses with stresses obtained for the existing conditions (during the time of observation) by computation or model testing." They had in mind that it is always necessary to analyse the results obtained by observation by the existing theories and this can only be done by using in the analysis conditions which existed at the time the observations were made. The worst conditions, which are usually those of the design, do not enable this comparison to be made, as in practice they very seldom occur.

With regard to the mounting of strain gauges there has been a great development in Portugal from the time when these instruments were first used.[3] Today the technique adopted is the one due to the Bureau of Reclamation which is doubtless a much better one than that used first.

The writers have not found the great difficulty mentioned by Mr. Xerez in discussing the influence of Poisson's ratio as they have been able to compare results obtained on equal models built of materials having different Poisson's ratio. These results have made it possible to supplement the ideas given in the paper, as they have shown that Poisson's ratio only has an appreciable influence in the stress field of the dam near the foundations. The principal maximum stresses increase a little there and the minimum ones are absolutely changed.

The statement that the comparison between the values observed on the prototype and the analytical results are not significant as neither the "trial load" method with only radial adjustments nor the independent arch method give "true results" for this type of dam merits consideration. The calculation methods are, like all theories, only approximate and never exact; theory is not in itself Nature. The comparison made is considered advantageous as these methods are used for design and it is necessary to show how near the "truth" they may be. It should be noted that it is precisely to this type of dam, more than any other, that they are usually applied.

An analysis of the stresses at all points of the dam was made with a view to seeing how far the differences could be explained by errors in reading. It was concluded that in many cases the errors were responsible for them, which is an important conclusion.

3. Serafim, J. Laginha—Measurement of Strains in Portuguese Concrete Dams—RILEM Symposium on the Observation of Structures, Lisbon 1955.

Discussion of "HYDRAULIC PRESSURE IN CONCRETE"

by T. C. Powers
(Proc. Paper 742)

T. C. POWERS.[1]—Professor Hrennikoff has presented a carefully reasoned argument about hydrostatic tension in water. Given the conditions and mechanisms that he visualizes, his position seems sound. But the conditions and mechanisms that he pictures cannot account for the phenomena we are trying to explain, and therefore it seems reasonable to suppose that some variation from his point of view is required.

Professor Hrennikoff expresses incredulity that the solid elements of a redwood tree could allow as much as 8 atmospheres of tension to develop in the sap of the tree. Yet, possibly he will agree that water is continually moved from the base to the top of tall trees, and therefore that the tree is somehow able to support a column, or, rather, innumerable columns, of water as high as it is. Perhaps this will not seem so incredible if one remembers that the tree has to support only the actual weight of the water, and that the stress in the solid part of the tree is not necessarily as great as the stress in the water. If the solid filaments are slender, we may be sure that the liquid filaments are even more slender. In fact, for liquid to be lifted much above 33 feet by transpiration, it must be in films so thin that bubbles of vapor cannot form in the films under the existing tension.

If water is held in a tube of ordinary dimensions, it can support very little tensile force. At relatively low stress, air or vapor bubbles appear and the column parts. If the water is free of dissolved air, tension can be considerable. A bubble of vapor will form when atmospheric pressure minus hydrostatic tension is less than about 25 mm of mercury, the pressure of the vapor in the bubble. However, if the water is in the form of a thin layer of filament, the bubble must form within this filament and must have a correspondingly small radius. The pressure on the bubble is equal to 1 atmosphere plus the pressure of liquid-surface tension minus hydrostatic tension in the liquid. Thus, at equilibrium,

$$p = A + \frac{2\sigma}{r} - t$$

where p = pressure of vapor in the bubble,
A = pressure of atmosphere on the liquid,
σ = surface tension of the liquid,
r = radius of bubble,
t = hydrostatic tension in the liquid.

At a given temperature, p has a fixed value; any pressure on a bubble that is higher than p will cause the vapor to condense and the bubble to disappear.

Ordinarily, the middle term on the right is negligible. But, if the filament

1. Basic Research Section, Portland Cement Assocn., Chicago, Illinois.

under tension is of the order 10^{-6} cm thick, the radius of a bubble in it must be less than half of that and then the middle term becomes very significant. Thus,

$$\frac{2\sigma}{r} = \frac{146}{0.5 \times 10^{-6}} = 292 \times 10^{6} \text{ dynes/cm}^2 ,$$

or about 300 atmospheres. In such a thin filament, a vapor bubble could not exist unless the hydrostatic tension exceeded 300 atmospheres.

The foregoing treatment of the subject will not stand rigorous scrutiny, but it at least suggests that the behavior of thin films of water is not the same as that of water in bulk.

The gist of the writer's discussion beginning on page 742-4 is that if the humidity at a given point inside the concrete of a dam is less than 100%,* hydrostatic pressure cannot be in existence at that point, and, it is to be expected that the humidity will be less than 100%* in most of a concrete dam. As to the latter point, a recent paper by Besson,[1] giving the results of measurements on a dam in Switzerland, showed that hydrostatic pressure did not exist in any part of the structure where measurements of moisture content were made, for at every point measured the moisture content was substantially less than that required for saturation. Thus, the condition postulated in the writer's discussion has been observed in a concrete dam.

It is a fact also that diminished humidity not only indicates absence of pressure but also existence of hydrostatic tension. This follows from the relationship between temperature and pressure, and the tendency to evaporate, i.e., the vapor pressure.** With a liquid under no pressure except its own vapor, adding hydrostatic pressure increases its vapor pressure; adding hydrostatic tension decreases it.

This does not dispose, of course, of the doubts expressed by Professor Hrennikoff about the reality of the hydrostatic tension and about the pressure differences that cause transmission through a dam exposed to water on one side and air on the other. It only shows that reduced humidity can indicate tension, and that reduced humidity has been observed, as predicted. We will now deal with tension more explicitly.

Consider a concrete specimen exposed to air on one side and water under pressure on the other, as in a permeability test. Consider that a steady state of flow has been established. Let us assume that the rate of evaporation from the downstream side exactly equals the rate of flow through the specimen. Under these conditions the concrete remains saturated throughout, and hydrostatic pressure exists at all points, as given by straight line A, Fig. 1.

By d'Arcy's law the rate of flow through the specimen is

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* Strictly speaking, 100% relative humidity exists when the vapor pressure of the solution in the pores of the concrete is equal to the vapor pressure of the same solution in bulk, at the same temperature.

** The vapor pressure of a liquid is the pressure of its own vapor when the liquid is in a closed space at specified temperature and hydrostatic pressure.

1. Besson, Marius, "Drying and Aging of Concrete in Dams," Schweiz. Bauztg. 72 (26) 371-75 (1954).

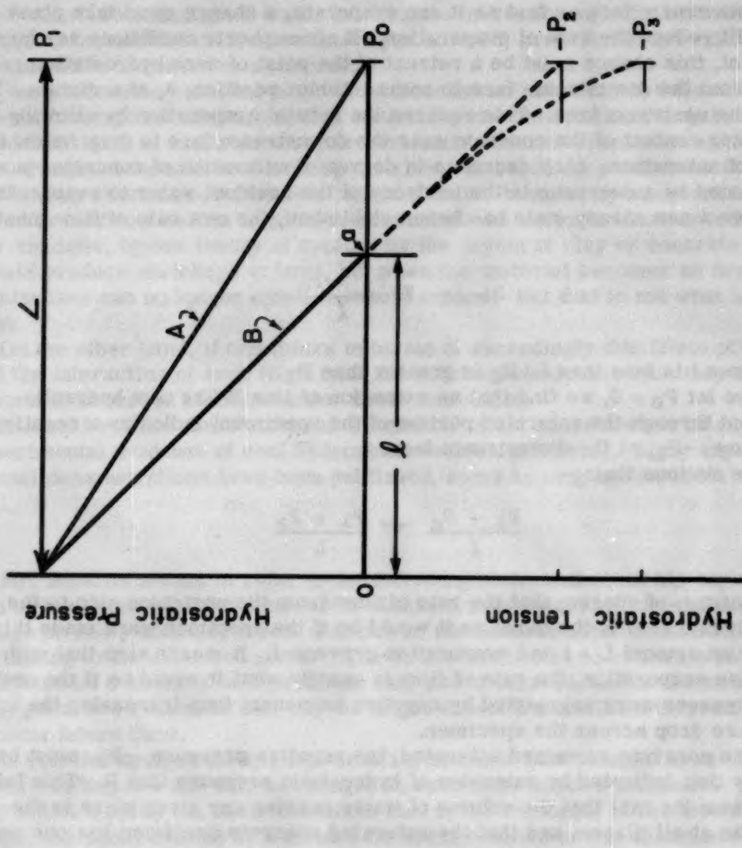
Hydrostatic Pressure
Hydrostatic Tension
P_1
P_0
$-P_2$
$-P_3$
L
ℓ
A
B
a
O

Fig. 1

$$R_1 = K\frac{P_1 - P_o}{L} \quad ,$$

in which R_1 = rate of flow through the specimen,
P_1 = hydraulic pressure at "upstream" face,
P_o = hydraulic pressure at "downstream" face,
L = thickness of concrete specimen,
K = coefficient of permeability of the concrete.

Keeping hydraulic pressure on the upstream face constant, let us increase the rate of evaporation by lowering the humidity of the surrounding air. (For simplicity, we assume isothermal conditions.) Since the rate of hydraulic flow under the original hydraulic gradient is not sufficient to deliver water to the downstream face as fast as it can evaporate, a change must take place that will reduce the rate of evaporation. If atmospheric conditions are kept constant, this change must be a retreat of the point of zero hydrostatic pressure from the downstream face to some interior position, a, at a distance l from the upstream face. This reduces the rate of evaporation by allowing the water content of the concrete near the downstream face to drop below the point of saturation. (Any decrease in degree of saturation of concrete is accompanied by a decrease in the tendency of the residual water to evaporate.)

After a new steady state has been established, the new rate of flow must be

$$R_2 = K\frac{P_1 - P_o}{l} \quad ,$$

and since l is less than L, R_2 is greater than R_1.

If we let $P_o = 0$, we find that an extension of line B (the new hydraulic gradient through the saturated portion of the specimen) indicates a negative pressure, $-P_2$, at the downstream face.

It is obvious that:

$$\frac{P_1 - P_o}{l} = \frac{P_1 + P_2}{L} \quad .$$

This means, of course, that the rate of flow from the upstream side to the downstream side is the same as it would be if the specimen were made thinner by an amount L - l and evaporation prevented. It means also that with effective evaporation, the rate of flow is exactly what it would be if the positive pressure were augmented by negative pressure, thus increasing the total pressure drop across the specimen.

If the concrete remained saturated, the negative pressure, $-P_2$, must be exactly that indicated by extension of hydrostatic pressure line B. This follows from the fact that the volume of water passing any given plane is the same as at all planes, and that the saturated concrete specimen has one and only one coefficient of permeability. But since, in fact, the concrete is progressively less saturated to the right of point a, its effective coefficient of permeability becomes progressively smaller in that part. Hence, to maintain a constant rate of flow, the hydraulic gradient must increase correspondingly, giving some such curve as the one terminating at $-P_3$. The

"simplified basis" mentioned on page 742-6 was simply that of ignoring this curvature and assuming that point a lay on a straight line between P_1 and $(-P_3)$. The resulting inaccuracy would be such as to give a position for point a too close to the upstream face.

It might be thought that to the right of point a, water flows as vapor, and thus notions of hydrostatic tension can be abandoned. Such an argument is untenable for the following reasons. In any case, flow must occur both as vapor and as adsorbed films, simultaneously. That part of the flow occurring in films must either be thought of as being due to hydraulic gradients in a region of hydrostatic tension, or else some new mode of thought about such flow must be supplied. That part that flows as vapor moves through a region of positive vapor pressure; there is no such thing as negative vapor pressure. In concrete, the part that flows as vapor must be negligible, as will be found when dimensions of pores are compared with the mean free path of vapor molecules. Practically all water in concrete moves through the adsorbed films. (Also meniscuses, as ordinarily conceived, probably do not exist in water held in hardened cement paste.)

The phenomena referred to in these discussions originate in the realm of submicroscopic structure, and this structure, and the relationship between solid and liquid in that structure, are evidently different from what some have imagined them to be. Holding the view that capillary phenomena are exclusively those associated with bodies of water bounded by meniscuses is bound to lead to a series of conclusions that do not conform to observed fact. For example, by one theory of capillarity the drying of clay or concrete should produce shrinkage at first, but when the material becomes so dry that meniscuses can no longer exist, it should expand. But that is not what happens.

On the other hand, if one thinks in terms of exceedingly thin films of water and the interaction of such films with the solid substrate on which they are adsorbed, observed facts no longer seem incredible.

It must be added that Professor Hrennikoff is not correct when he says experimental evidence of real hydrostatic tension is lacking. Many experimental demonstrations have been published, some as long ago as 1850.

* * *

Mr. Serafim seems to refer to the following statement about his explanation of the relatively low-area factor obtained with water: "This explanation, though possibly correct, seems doubtful on the basis of other considerations pertaining to the adsorbed state." The writer would like to reiterate his belief that Mr. Serafim's interpretation may prove correct, and at the same time point to a reason or two why we might expect a modified interpretation at some future time.

In calculating area factor, Mr. Serafim assumes that changes in extensometer reading are due only to changes in externally applied pressure. This may not be exactly so. Let us assume that at the beginning of the test with water, the alkali content of the concrete specimen was evenly distributed. During the test, fresh water enters the central sector and the original solution is displaced toward the ends, some of it appearing at the exposed surface where it may become concentrated by evaporation. This produces a gradient in alkali concentration which, by osmosis, will tend to extract water from the central region where the alkali concentration is lowest. This tendency should

affect the indicated strain in the central sector where the extensometer is; that is, the extensometer would indicate shrinkage, and this effect would be independent of the magnitude of the applied pressure; it would depend only on osmotic pressure. The effect on calculated area factor might or might not be significant, depending on the relative deficiency of alkali in the central sector. The point is that the magnitude of the effect of osmotic pressure should be known and, if necessary, taken into consideration in the calculation of area factor.

Another basis for questioning Mr. Serafim's interpretation is that there is reason to believe that adsorbed water is not solid. It should flow under any stress, however small, and thus it should transmit pressure. However, we have recently obtained data indicating that the first two or three layers of adsorbed water behaves as if they have very high viscosity. This viscosity may be so high that the layers behave practically (though not absolutely) as if they were solid, and, if so, Mr. Serafim's interpretation may be correct.

On the other hand, if this interpretation is correct, it raises a question about the correctness of the values found with nitrogen. The conditions under which the specimens were dried were such as to leave one or two adsorbed layers in the specimen. If these layers behaved as solids under the water test, they should have behaved the same way in the nitrogen test and, therefore, the nitrogen test should have given the same result as the water test.

It seems significant that when testing granite, Mr. Serafim obtained the same results with water and nitrogen. In granite the effects of osmotic pressure or of internal cracking should be negligible. However, it is true also that the effect of a practically immobile adsorbed layer should be negligible because the ultimate particles of the granite are much larger than the gel particles of cement paste, and the role of adsorption is correspondingly much smaller.

* * *

The point of view taken by Dr. Leliavsky in his various discussions of the uplift question is, in many respects, the same as the writer's; indeed, some of the writer's views are undoubtedly derived from Leliavsky's writings. But it seems that a considerable part of his discussion of the writer's present paper is more relevant to other papers than to this one. The writer has not been concerned, for example, with the relative merits of different experimental means of measuring area factors. Some of Dr. Leliavsky's comments bear on the final report of the Subcommittee on Uplift in Masonry Dams of ASCE, for which the writer is not at all responsible. Mr. Ross Riegel, formerly chairman of that subcommittee, has appended to the writer's remarks a reply to those portions of Mr. Leliavsky's discussion which concern the subcommittee.

According to Dr. Leliavsky, the paper outlines what the writer believes to be an "adequate conclusion." From the context, it is surmised that Dr. Leliavsky means the writer has reached the conclusion that the Subcommittee sought but did not reach. It is to be hoped that the Subcommittee did not so construe the writer's remarks.

The writer is not certain as to what part of his paper Dr. Leliavsky referred when he spoke of an "adequate conclusion," but he seems to believe that the writer concluded that each particle of gel is bonded to its neighbors at exactly 12 points, that each point of bonding has a certain known area, that

the area factor of each particle is 99.8%, and that the area factor of each particle is identical with that of concrete. But such an interpretation can hardly be justified in view of the paragraph on page 742-3 which says, "The foregoing discussion does no more than arrive at an estimate of a possible upper limit of an area factor. The actual area factor is obtainable only by experiment."

The intended implication was that experimental results, giving a high area factor, such as those reported by Serafim or Leliavsky, could be considered compatible with what is known about the structure of paste. That knowledge indicates that the gel particles are "spot welded" to each other, and that the total area welded, and therefore unwettable, might be only a small fraction of the total particle area, and that the area factor for the whole concrete cannot be less than that for the gel particles.

The writer cannot agree that information on "microstructure" of concrete is irrelevant. The ultimate particles with which water makes contact in concrete are exceedingly small and numerous—they are not much larger than molecules. That very fact shows that irregularities in their spatial distribution, and even irregularities among them as to physical characteristics, could not give rise directly to vagaries in test results. Reproducibility of an area factor, which seems anomalous to Dr. Leliavsky, should be expected if the effective stresses are those that arise from hydraulic pressure on the gel particles, or, as Dr. Leliavsky prefers, hydraulic pressure in the microstructure. Arguments based on drawings like Figs. 2a and 2b seem unnecessary and misleading. The geometry indicated in those figures is hardly conceivable in a real, permeable solid.

ROSS M. RIEGEL,[1] M. ASCE.—In his discussion of Mr. Powers' Paper, Mr. Leliavsky makes certain criticisms of the report of the Subcommittee on Uplift, which in the writer's opinion call for some comments.

The members of the Subcommittee had access to a preliminary memorandum on this subject by Mr. Powers about 1949, and considered incorporating his conclusions in its Report. Mr. Powers at that time was not encouraging, and the writer understood that he wanted to check his ideas further before getting into print. (He may have preferred to present his conclusions to the Profession in person, as he has done, and the writer would have no criticism of this attitude.) Since Mr. Leliavsky appears to disagree with some of Powers' presentation, it seems that the behaviour of concrete from the standpoint of internal hydraulic pressure is still in a somewhat speculative and controversial stage, and that the Subcommittee was warranted in steering away from definite conclusions thereon. Through one of its members, the Subcommittee was warranted in steering away from definite conclusions thereon. Through one of its members, the Subcommittee was aware of the program of investigations reported by Mr. Carlson in Paper No. 700 on "Permeability, Pore Pressure and Uplift in Gravity Dams," although the results of the same could not be foreseen.

As the writer read the present paper, much of the matter seemed familiar. To him, it appeared convincing as it did when first seen in 1949. It confirms the idea that uplift in the foundation is the main concern of the dam designer

1. Cons. Engr., Gibbs & Hill, Inc., New York, N. Y. Formerly, Chairman, Subcommittee on Uplift in Masonry Dams.

and that when the concrete is provided with drainage wells, as is recommended practice, one can well forget about uplift effects above the foundation. The Subcommittee's concentration on the foundation seems to be justified.

Mr. Leliavsky further complains that the Subcommittee failed to recognize certain "advanced mathematical solutions." Most of the Subcommittee were consulting and designing engineers who sought to develop practical conclusions which would facilitate design. The members had wide experience in design and construction and had ripe judgment. Two of them were Honorary members of the Society. The Subcommittee was fitted to review such existing data as seemed pertinent, rather than to conduct mathematical investigations. Nor had it any funds for research. If it had ventured into these fields, it might have been even now struggling to prepare a report. (It had been in existence about six years). Meanwhile, two valued members have died, Messrs. Creager and Harza, two more are past eighty, and the rest are growing no younger. It seems to the writer that the Subcommittee as it was constituted, did about as good a job as could be expected of it. It specifically suggested the creation of a new Committee to consider evidence on concrete conditions, as and when the same should become available.

Discussion of "ARCH DAMS: THEIR PHILOSOPHY"

by Andre Coyne
(Proc. Paper 959)

G. S. SARKARIA,[1] J.M., ASCE.—That the arch dam, though ancient in concept, is yet in an active stage of development at the present time is the main theme of this interesting paper by M. Coyne. Studying this and the other papers presented at the A.S.C.E. Symposium on Arch Dams, June 1956, the writer is impressed by the diversity of methods of design, analysis, and experimentation adopted by various engineers and organizations for obtaining arch dams most suited to their requirements.

All arch dams have one property in common: curvature. Otherwise there appear to be numerous types of arch dams which differ from each other substantially in their structural and shape characteristics. M. Coyne, for example, has named these different types in his paper: "very thin arch," dome shaped or overhanging arch, arch sloping downstream, "long but low dam, with two hinges," and multiple arch. Other authors have used terms like single-curvature(1) and double-curvature(2) arch dams. Terms like arch-gravity, variable-thickness arch and constant-angle arch dams are of more common usage. There are other arch dams with ungrouted radial contraction joints, an ungrouted "perimetral joint,"(3) or a "horizontal sliding joint."(4)

The writer believes it is desirable to classify various types of arch dams in order to appreciate the different designs discussed by M. Coyne and other contributers to the Symposium. The two major classifications are:

1. Shape Classification: Terms such as constant-thickness, variable-thickness and constant-angle refer to the shape of the arch. Other designations such as single-curvature, double-curvature, dome or overhanging type signify the shape characteristics of the dam in two directions.

2. Structural Classification: Arch dams with horizontal or peripheral sliding or keyed joints, and those with radial contraction joints keyed and/or grouted, represent distinctly different structural types.

It is necessary to adopt a structural classification that will clearly distinguish between different types of structures. The examination of merits and suitability of the different structural types to a particular situation should form as much a part of design and analyses as the adoption of the final arch dimensions. The writer proposes the following preliminary structural classification:

a. Monolithic Arch Dams: This type includes arch dams in which all contraction joints, whether horizontal, vertical, or peripheral are grouted.

b. Simple Arch Dams: Arch dams that are designed as horizontal elastic arch slices fixed at the abutments and where a horizontal sliding joint is provided between the dam and its base. Such dams are not three-

1. Engr., International Eng. Co., San Francisco, Calif.

dimensional monolithic structures, since bending moments can not be transferred across the sliding joint to the base of the dam. Matilija Dam[4] in California is an example of this type.

c. Hinged Arch Dams: Arch dams that have vertical (radial) contraction joint that are keyed but not sealed with grout, are hinged structures. Dams with peripheral hinged joints, such as Osiglietta Dam[3] in Italy, should also be classified as hinged arch structures. The main advantage[3] claimed for hinged arch dams is, "the stresses in the structure were lower and better distributed."

d. Cantilever Arch Dams: Such arch dams have radial (vertical) contraction joints that are neither keyed nor grouted. The arch voussoirs are vertical cantilevers that wedge against each other when subjected to reservoir water pressure.

M. Coyne has mentioned that progress in the design of arch dams is dependent upon factors like shape of the dam-site, stiffness of the arch, the allowable average stresses, and the quality and performance of concrete. Comparing the various designs of arch dams described in the papers submitted for the Symposium, the writer has no doubt that many other engineers like himself must be asking the question "Why is there so much difference between the various designs?" The writer feels that an analysis of the factors that influence the design of arch dams should help to answer the above question to some extent. Some of these major factors are:

a. Shape of canyon at dam-site.
b. Structural type of arch dam.
c. Height of the structure.
d. Forces acting against the dam, and the competence of the foundation and abutments.
e. Allowable stresses.
f. Methods of design and analysis.

These factors are discussed in brief in the following paragraphs.

Canyon Shape at Dam-site:

It has long been a matter of common belief amongst dam designers that economical arch dams cannot be built at a site where the crest length to maximum height ratio exceeds 5. M. Coyne states that the use of structural models "has enabled us to go further and further in the use of wide valleys—even valleys wide at their bases—without any fear of excessive deflections . . . ." The design of an arch dam is affected not only by the crest length to height ratio but also by the shape of the canyon and length of the peripheral contact. In order to systematically investigate influence of canyon shape, it is necessary that a standard classification of canyon shapes is referred to by all designers. In collaboration with Mr. F. D. Kirn, the writer has proposed[5] a classification, which if adopted, should eliminate the confusion caused by references to general terms like wide, narrow, and U-shaped canyons. This classification is shown in the writer's Fig. (1).

An effort was also made to establish a simple yet appropriate criterion based on shape of canyon at a dam-site, that would indicate the suitability or otherwise of the site for an arch dam. A "canyon-shape factor" defined as the ratio of the foundation and abutment perimeter to the maximum height of the dam, was therefore proposed.[5] Refering to the writer's Fig. (2), the

canyon-shape factor is:

$$K = \frac{b + H (\sec \psi_1 + \sec \psi_2}{H}$$

The notations are explained in the sketch.

Two dam-sites having the same crest-length to height ratio are likely to have different shape characteristics. The two canyons shown in the writer's Fig. (3) have the same B to H ratio, but Profile I and II have canyon-shape factors of 4.5 and 5.3 respectively. The difference between the canyon-shape factors indicates that an arch dam for Profile II would be much more massive than the one for Profile I, provided both the dams are designed according to the same criteria and by the same method.

To illustrate this point further, canyon profiles and crown cantilever sections of Hungry Horse Dam and the proposed Yellowtail Dam(5) are shown in the writer's Fig. (4). Hungry Horse site has a wide V profile and its canyon-shape factor computed from the developed profile is 4.6. Yellow tail dam-site has a composite U-V shaped canyon with comparatively steeper abutment slopes, and its canyon-shape factor is 3.5. Both these dams were designed by the U. S. Bureau of Reclamation using the trial load method. The loading conditions and design criteria were reasonably similar. The influence of canyon shape is dramatically evident from comparison of the crown cantilever sections for the two dams.

Canyon-shape data for 20 dams is presented in the writer's Table I. It includes both arch and straight-gravity dams. For arch dams developed profiles along arch center lines were used in obtaining canyon-shape factors. The writer has also included some of the dams described by M. Coyne and other contributers to the Symposium. Undoubtedly some of the data are approximate as sufficient details are not available to the writer.

Comparing the canyon-shape factors for the arch and gravity dams listed in Table I, it should be noted that for arch dams, canyon-shape factors vary between 2.2 and 4.6, the average being 3.4. Canyon-shape factor for Kariba Dam proposed by M. Coyne is 6. Since the site for this dam is exceptionally wide compared to those conventionally considered suitable for arch dams, its canyon-shape factor is not considered representative. Canyon-shape factors for most of the gravity dams are well above 6. If any gravity dams have been built in canyons with shape factors less than 5, it is presumed that considerations other than shape of site profile must have disfavoured the adoption of an arch dam.

From this empirical comparison it can be concluded that sites with canyon-shape factors greater than 5 are unsuitable for building economical arch dams. The author, however, proposes arch dams for sites which would be considered too wide by most designers and may, therefore, change the above empirical limit of canyon-shape factor for arch dams.

Structural Types and Height:

The writer has already briefly discussed the desirability of classifying arch dams according to their designed and built-in structural characteristics. Height of dam is also a common denominator in all types of arch dams, and all schools of thought feel that arch dams are the type that can be economically built to heights for which gravity or earth may not be feasible.

Methods of Design, Allowable Stresses, and Forces Acting on Dam:

The author has mentioned, although briefly, three or four current design concepts and methods. He shares with many other designers the dread of "extreme complexity" of elaborate analytical methods. So he suggests that

> "the best thing to do is to put simplified calculations, giving results within two limits and to get from them the courage for making up one's mind for choosing a given design. This is then immediately elaborated on a model. . . Today, over-exacting calculations, should they be necessary, particularly for calming the troubled consciences of a few people, are undertaken as a final check, after all the dimensions have been established in the laboratory."

The writer finds this statement rather paradoxical. M. Coyne does not discredit "over-exacting" analytical calculations as incorrect, for to satisfy "some troubled consciences," if necessary, he does consider analytical methods good enough to check the finished product of laboratory experiments. Without meaning to minimise the importance of structural models, the writer believes it is necessary to deprecate the tendency amongst many designers to reject detailed analytical analyses as "lengthy," "cumbersome," time-consuming, and expensive.

Arch dam design will greatly benefit if reasonably exact analytical methods and not too "over-exact" structural model tests compliment each other. After all, analytical methods and scale models often work under similar, though occasionally unrealistic, design assumptions. Analytical methods can often be modified and shortened within similar limits of accuracy, to be economically comparable with model tests.

The controversy as to which method of design is better can be detected as an undercurrent in many of the papers submitted to the Symposium. Often, it is not the method of design, but the design forces, allowable stresses, and the quality of concrete that determine the section finally adopted for an arch dam. Some design criteria require that no tensile stresses should occur in the structure, anywhere. Describing the very thin Le Gage Dam, M. Coyne states "in some places the concrete freed itself from excessive extension by cracking, without any serious consequences." Also, "experience proves that extension and even cracks in arch dams are not dangerous." Evidently it is a matter of judgment as to how much tension and cracking can be safely allowed in an arch dam, and on this may depend the adopted thickness of the dam.

M. Coyne also points out the improvement in allowable average stresses in concrete over a period of years. This is another factor that should not be lost sight of when comparing two dams designed by different methods. For example, it would not be correct to compare the sections of Hungry Horse Dam(6) where the maximum allowable compressive stress is 750 lb/in^2 against those for Malpasset Dam for which the allowable stresses are of the order ot 850 to 1000 lb/in^2 as mentioned by the author. All other factors being comparable, this disparity in allowable stresses alone would indicate a proportional difference in the sections.

Having described some of the factors that should not be ignored in comparing various types of arch dams and methods of design, the writer would like to express the wish that in appreciation of the healthy arguments and spirit of rivalry generated by the papers submitted by M. Coyne and others, dam designers would evolve design concepts and criteria that are a compromise between the various points of view and thus useful and acceptable to apparently opposing viewpoints.

TABLE I

CANYON-SHAPE CHARACTERISTICS OF MASONRY DAMS

No.	Dam	Location	Max Height in feet	Shape of Canyon	Canyon-shape Factor K	Remarks (Profile)
(a) ARCH DAMS						
1.	Arrowrock	U.S.A.	350	Composite U-V	4.3	Symmetrical
2.	Buffalo Bill	U.S.A.	325	U	2.2	
3.	Cabril	Portugal	442	Wide V	3.5	Unsymmetrical
4.	Castelo Do Bode	Portugal	378	Composite U-V	3.4	
5.	Hoover	U.S.A.	726	Composite U-V	2.5	
6.	Hungry Horse	U.S.A.	564	Wide V	4.6	Unsymmetrical
7.	Kamishiiba	Japan	370	Wide V	3.9	
8.	Kariba	Africa	400	Composite U-V	6.0	Proposed
9.	Ponte Racli	Italy	164	Wide V	3.1	
10.	Ross	U.S.A.	655	Wide V	3.5	Final stage height
11.	Santa Giustina	Italy	495	U	2.4	Symmetrical
12.	Tumut Pond	Australia	265	Wide V	3.2	Proposed
13.	Venda Nova	Portugal	318	Wide V	3.5	Unsymmetrical
14.	Yellowtail	U.S.A.	520	Composite U-V	3.5	Proposed
(b) STRAIGHT GRAVITY DAMS						
15.	Bhakra	India	680	Wide V	3.4	Under construction
16.	Fontana	U.S.A.	480	Composite U-V	5.2	
17.	Friant	U.S.A.	319	Wide and flat	12.2	
18	Grand Coulee	U.S.A.	510	Wide and flat	9.5	
19.	Kortes	U.S.A.	240	Composite U-V	3.0	
20.	Norris	U.S.A.	265	Wide V	3.0	

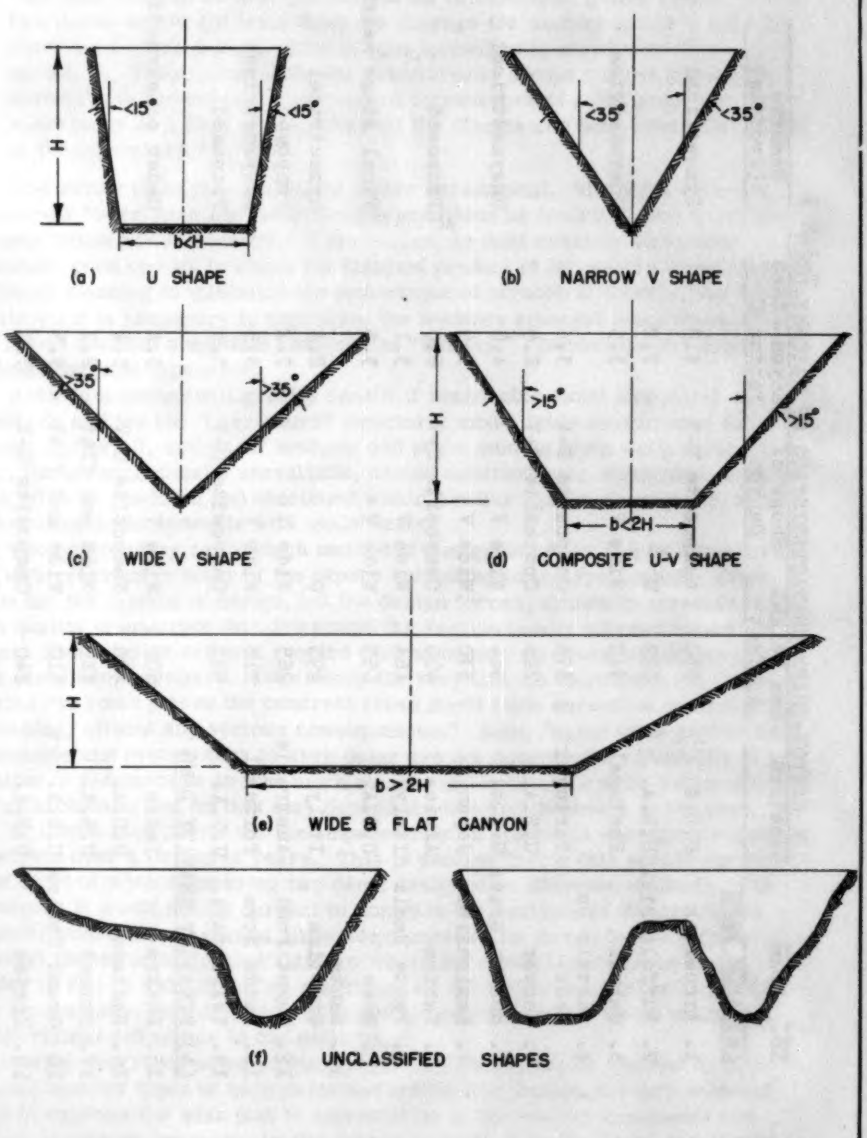
H
<15°
<15°
b<H
(a) U SHAPE
<35°
<35°
(b) NARROW V SHAPE
>35°
>35°
(c) WIDE V SHAPE
H
>15°
>15°
b<2H
(d) COMPOSITE U-V SHAPE
H
b > 2H
(e) WIDE & FLAT CANYON
(f) UNCLASSIFIED SHAPES

FIG. 1

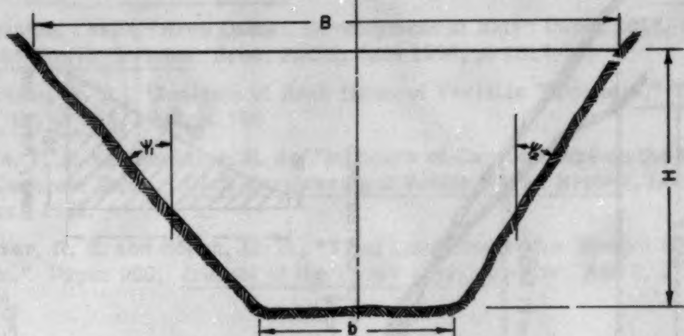
B
H
b
ψ_1
ψ_2

FIG. 2

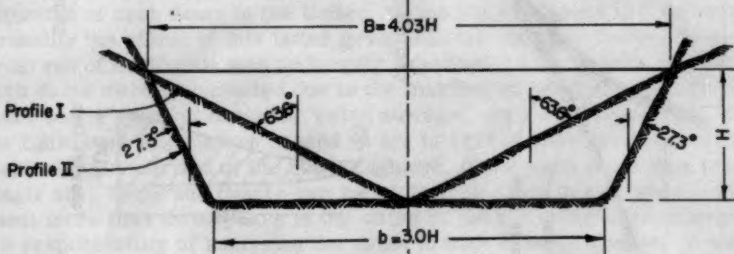
B=4.03H
H
Profile I
Profile II
63.6°
63.6°
27.3°
27.3°
b=3.0H

FIG. 3

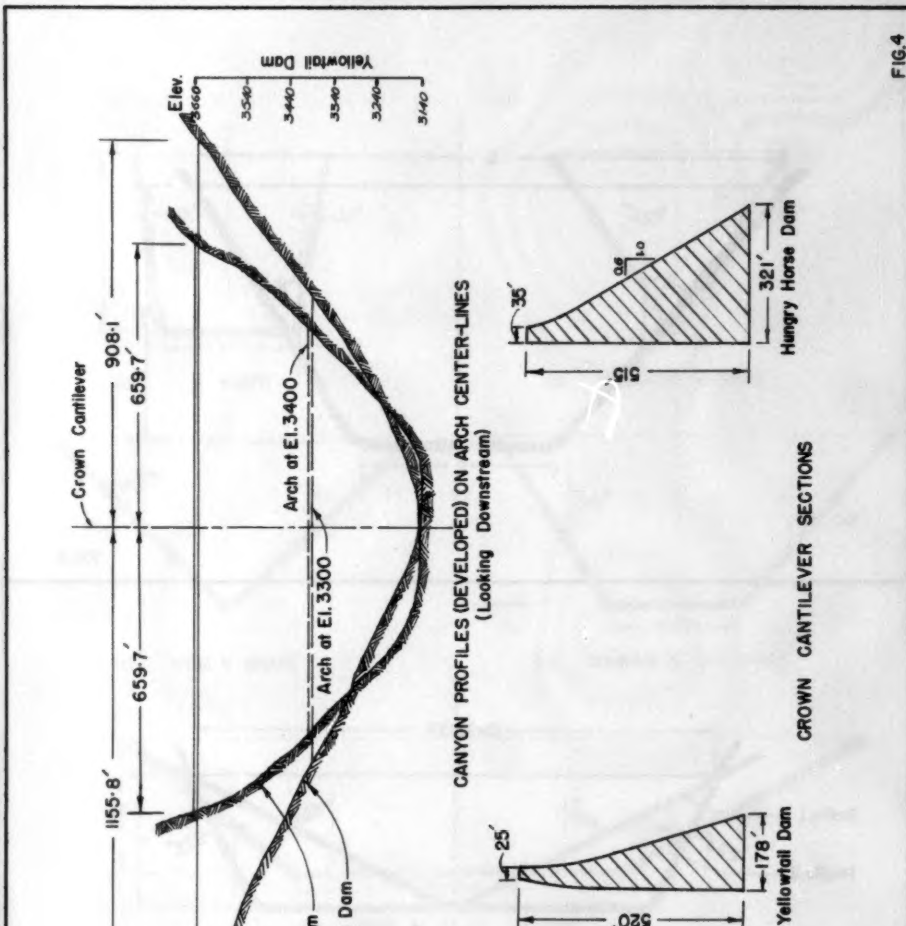
Crown Cantilever
908·1′
659·7′
1155·8′
659·7′
Elev.
Yellowtail Dam
3660
3540
3440
3340
3240
3140
Elev.
Hungry Horse Dam
3565
3450
3350
3250
3150
3050
Arch at El. 3400
Arch at El. 3300
Yellowtail Dam
Hungry Horse Dam
CANYON PROFILES (DEVELOPED) ON ARCH CENTER-LINES
(Looking Downstream)
35′
0.6
1.0
515′
321′
Hungry Horse Dam
25′
520′
178′
Yellowtail Dam
CROWN CANTILEVER SECTIONS

FIG. 4

REFERENCES

1. Marcello, Claudio, "Santa Giustina Single-Curvature Arch Dam." Paper 992, Journal of the Power Division, Proc. ASCE, June 1956.

2. Marcello, Caludio, "Isolato Double-Curvature Arch Dam." Paper 995, Journal of the Power Division, Proc. ASCE, June 1956.

3. Semenza, Carlo, "Arch Dams: Development in Italy" Paper 1017, Journal of the Power Division, Proc. ASCE, June 1956, p. 1017-36.

4. Perkins, W. A., "Analysis of Arch Dams of Variable Thickness," Trans ASCE Vol. 118, 1953, p. 766.

5. Kirn, F. D. and Sarkaria, G. S., "Influence of Canyon Shape on the Design of Concrete Dams," Civil Engineering & Public Works Review, London, March 1955.

6. Glover, R. E. and Copen, M. D., "Trial Load Studies for Hungry Horse Dam." Paper 960, Journal of the Power Division, Proc. ASCE, April 1956.

GEORGE E. GOODALL,* M. ASCE.—The author properly begins his very interesting paper with a brief mention of 16th Century arch dams in Spain and Ponte-Alto Dam in Italy. It seems probable that the earliest origin of arch dams reaches still further back in history. About fifteen years ago, the writer encountered an article in "Irrigacion en Mexico" in which it was stated that arch dams had been constructed by the Arabs in Africa before the Christian Era.

The development of arch dam designs in the United States, as well as in other countries, has undergone a constant evolution. A large part of the construction of arch dams in the United States has been done in California. Probably the oldest of this latter group was the old Bear Valley Dam. As the progress of California was so largely dependent on irrigation, a great many arch dams were constructed due to the inability of local interests to finance more costly types of dams for water storage. As a consequence, by the time the California Legislature passed an act in 1929 placing all dams in the State under the jurisdiction of the State Engineer, there were more than eighty single arch dams and twenty-two multiple arch dams in existence. The writer spent more than three years in the office of the State Engineer, charged with the responsibility of analyzing the dams in both of these groups. It was evident that older arch dams had been analyzed solely by the "cylinder" formula. The engineers responsible for these designs, realizing the shortcomings and inaccuracies of the "cylinder" formula, limited "cylinder" stresses to about 300 p.s.i. With the aid of the Cain formulae,[1] augmented by the later work of B. F. Jacobson,[2] Dr. Vogt,[3] and others, newer designs were made in which the "cylinder" formula was used only for a rough preliminary with the result that higher cylinder stresses became the rule.

In the early days of the California State Supervision of Dams, maximum

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* Cons. Civ. Engr., Sacramento, Calif.
1. Trans. ASCE, Vol. LXXXV, p. 233.
2. Trans. ASCE, Vol. 90, p. 475.
3. Trans. ASCE, Vol. 93, p. 1272.

arch stresses, as computed by the Cain Formulae, were limited to 600 p.s.i. in compression, but tensile stresses not to exceed 100 p.s.i., even though occuring at contraction joints, were permissible. In more recent years, these limitations have been modified and in the case of Donnell's Dam, now under construction on the Stanislaus River, California, the maximum compressive stress, as calculated by the Vogt Formulae, is 850 p.s.i., including the effects of a 20 degree F. temperature drop. In adapting the Vogt Formulae for this work, the modulus of elasticity of the abutment rock was taken as 1.5 times the modulus of elasticity of the concrete. Inasmuch as the effect of abutment deformation is negligible for arches having a small ratio of t/r, the arches in the upper portion of the dam would show the highest stresses as computed by the "cylinder" formula. In this specific instance the maximum "cylinder" stress is 766 p.s.i., 77' below crest of dam. From this elevation down toward the base, "cylinder" stresses progressively decrease. Donnell's Dam,[4] when completed, will be 485' high and due to the great economy of the arch dam, will cost slightly under 10-million dollars.

The author's statement that "because an arch is curved, the difficulty would seem to be not to make it hold up, but to knock it down" is not surprising to anyone who has had considerable experience with many of the existing older arch dams. The old Bear Valley Dam and Upper Otay Dam, both in California, which have long been considered classic examples of boldness, would seem to have had considerable influence in the author's statement quoted above. Two of the older arch dams in California had arches that were not even circular in plan but rather roughly in the shape of a spiral. They have no axis of symmetry although the loads imposed thereon are symmetrical. However, they have been in service for many years. Another example, if any be needed to back up the author's statement, is one arch dam 178' in height, whereat the actual excavation contours for the lower half of the height of the dam diverge in a down stream direction. Yet this dam in the disasterous flood of December, 1937, was overtopped by 16.2' and did not fail.

Figure 1 shows plan and abutment details of an arch dam in California approximately 50' in height which has no abutment whatsoever at the right end of the arch. The unbalanced "cylinder" thrust at the end of the arch is 4,650,000 pounds. The spillway is inadequate and the entire arch has been overtopped, yet this structure still stands.

The writer does not mention these obviously horrible examples as something that should be repeated or even allowed to influence design in any way. These have been mentioned merely to reinforce, if any such reinforcement be needed, the author's contention that the arch inherently is a very safe structure. His reference to an experimental dam model which failed under "cylinder" stress of 4300 p.s.i. adds further weight to the reported test data on models performed at Lake Cushman Dam.[5]

Where topographic and geologic considerations are favorable, there can be no question as to the economy of the arch dam. In the case of Donnell's Dam, when the site was first explored, a competent geologist estimated that the depth of alluvial fill in the stream channel at the site would be in excess of 75'. Preliminary designs and estimates showed that a rock fill dam would cost in excess of $1,000,000 more than a concrete arch. Diamond drilling of the site showed that the depth of the alluvium was about 200' where the arch

4. Engineering-News Record, August 16, 1956, p. 42.
5. Trans. ASCE, Vol. 90, p. 553.

dam crossed the channel section. It is obvious that excavation through this depth of alluvium for any type of dam other than a thin arch dam, would have been so costly as to have rendered the entire project uneconomical.

Much has been written in the past thirty-five years about the calculation of stresses in an arch dam. The farther one goes, the more complex the analyses become and the claims of accuracy of stress determination seem to grow in direct ratio to complexities of the mathematics involved. It has been well stated by Dr. Fredrik Vogt that "a rough approximation based on correct assumptions with regard to shrinkage, temperature changes, yielding of foundation, etc., is more valuable than a highly refined computation based on incorrect assumptions.[6] In none of the papers expounding methods of stress determination has the writer even seen an attempt to calculate temperature stresses in the arches of a dam which considers the effect of the non-linear temperature variations experienced throughout the thickness of any arch ring such as those measured at the Englebright (Upper Narrows) Dam.[7] Concrete placement in this dam was completed in December 1940 and the reservoir filled to spillway elevation in five days. At the end of the filling period, the Carlson strain meters embedded near the faces of the dam at the abutments in general indicated no change in stress between the conditions of no load and full load. Some years later, after the complete dissipation of the heat of hydration of the cement, measured stresses began to follow definite patterns. The meters adjacent to the up stream face were fairly stable as far as temperature was concerned and followed the center temperature. Temperatures near the down stream face varied throughout wide ranges with the atmospheric temperature, but showed a time lag with increase in depth from the face of dam. The time lag mentioned above refers to the diurnal temperature variations. The arch ring 118' below spillway has a thickness of 59.25'. For this arch ring the time lag of seasonal temperature variation appears to be about four months. After complete dissipation of the heat of hydration, maximum temperatures at the center line of the arch ring were observed to be about December 1, and minimum temperatures about May 15. The termperature 1' from the down stream face could vary any where from below 50 degrees to in excess of 90 degrees. As a result of these temperature variations, the measured stresses showed considerable variation from the calculated stresses and frequently the sign of the measured stress reversed from that of the calculated stress.

The writer has made inumerable and voluminous stress analyses of many arch dams. It is certain that many indeterminate factors can not be evaluated. Two of the most important indeterminate factors that can not be evaluated, either in stress calculations or model studies, are the non-linear temperature variations refered to above and the effect of contraction joints, grouted or not grouted. It would seem fitting to refer to conclusion to the closing statement of the late William Cain, M. ASCE, in his discussion of B. F. Jacobson's classic paper "Stresses in Thick Arches of Dams" where he wrote, "He seems well aware that an exact solution of the arch dam is not to be looked for, so that all an engineer can do is to examine the various influences and combine them to effect a practical solution."[8]

6. Trans. ASCE, Vol. 93, p. 1272.
7. Journal of the American Concrete Institute, September 19, 1947, p. 65.
8. Trans. ASCE, Vol. 90, p. 547.

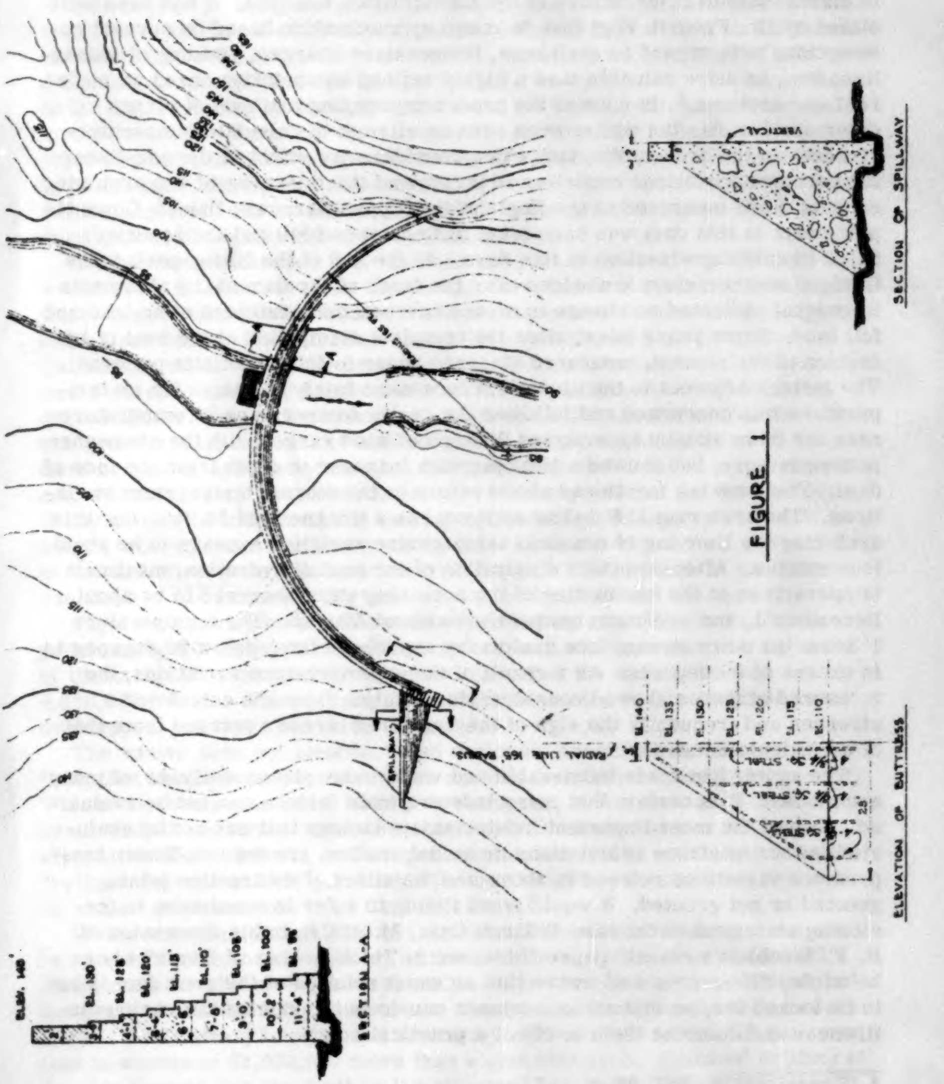
SECTION OF SPILLWAY
VERTICAL
ELEVATION OF BUTTRESS
SECTION A-A
SPILLWAY

FIGURE 1

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW) divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 861 is identified as 861 (SM1) which indicates that the paper is contained in issue 1 of the Journal of the Soil Mechanics and Foundations Division.

VOLUME 81 (1955)

OCTOBER: 809 (ST), 810 (HW)[c], 811 (ST), 812 (ST)[c], 813 (ST)[c], 814(EM), 815(EM), 816(EM), 817(EM), 818(EM), 819(EM)[c], 820(SA), 821(SA), 822(SA)[c], 823(HW), 824(HW).

NOVEMBER: 825(ST), 826(HY), 827(ST), 828(ST), 829(ST), 830(ST), 831(ST)[c], 832(CP), 833(CP), 834(CP), 835(CP)[c], 836(HY), 837(HY), 838(HY), 839(HY), 840(HY), 841(HY)[c].

DECEMBER: 842(SM), 843(SM)[c], 844(SU), 845(SU)[c], 846(SA), 847(SA), 848(SA)[c], 849(ST)[c], 850(ST), 851(ST), 852(ST), 853(ST), 854(CO), 855(CO), 856(CO)[c], 857(SU), 858(BD), 859(BD), 860(BD).

VOLUME 82 (1956)

JANUARY: 861(SM1), 862(SM1), 863(EM1), 864(SM1), 865(SM1), 866(SM1), 867(SM1), 868(HW1) 869(ST1), 870(EM1), 871(HW1), 872(HW1), 873(HW1), 874(HW1), 875(HW1), 876(EM1)[c], 877 (HW1)[c], 878(ST1)[c].

FEBRUARY: 879(CP1), 880(HY1), 881(HY1)[c], 882(HY1), 883(HY1), 884(IR1), 885(SA1), 886(CP1), 887(SA1), 888(SA1), 889(SA1), 890(SA1), 891(SA1), 892(SA1), 893(CP1), 894(CP1), 895(PO1) 896(PO1), 897(PO1), 898(PO1), 899(PO1), 900(PO1), 901(PO1), 902(AT1)[c], 903(IR1)[c], 904 (PO1)[c], 905(SA1)[c].

MARCH: 906(WW1), 907(WW1), 908(WW1), 909(WW1), 910(WW1), 911(WW1), 912(WW1), 913 (WW1)[c], 914(ST2), 915(ST2), 916(ST2), 917(ST2), 918(ST2), 919(ST2), 920(ST2), 921(SU1), 922(SU1), 923(SU1), 924(ST2)[c].

APRIL: 925(WW2), 926(WW2), 927(WW2), 928(SA2), 929(SA2), 930(SA2), 931(SA2), 932(SA2)[c], 933(SM2), 934(SM2), 935(WW2), 936(WW2), 937(WW2), 938(WW2), 939(WW2), 940(SM2), 941 (SM2), 942(SM2)[c], 943(EM2), 944(EM2), 945(EM2), 946(EM2)[c], 947(PO2), 948(PO2), 949(PO2), 950(PO2), 951(PO2), 952(PO2)[c], 953(HY2), 954(HY2), 955(HY2)[c], 956(HY2), 957(HY2), 958 (SA2), 959(PO2), 960(PO2).

MAY: 961(IR2), 962(IR2), 963(CP2), 964(CP2), 965(WW3), 966(WW3), 967(WW3), 968(WW3), 969 (WW3), 970(ST3), 971(ST3), 972(ST3)[c], 973(ST3), 974(ST3), 975(WW3), 976(WW3), 977(IR2), 978(AT2), 979(AT2), 980(AT2), 981(IR2), 982(IR2)[c], 983(HW2), 984(HW2), 985(HW2)[c], 986(ST3), 987(AT2), 988(CP2), 989(AT2).

JUNE: 990(PO3), 991(PO3), 992(PO3), 993(PO3), 994(PO3), 995(PO3), 996(PO3), 997(PO3), 998 (SA3), 999(SA3), 1000(SA3), 1001(SA3), 1002(SA3), 1003(SA3)[c], 1004(HY3), 1005(HY3), 1006 (HY3), 1007(HY3), 1008 (HY3), 1009 (HY3), 1010 (HY3)[c], 1011 (PO3)[c], 1012 (SA3), 1013 (SA3), 1014(SA3), 1015(HY3), 1016(SA3), 1017(PO3), 1018(PO3).

JULY: 1019(ST4), 1020(ST4), 1021(ST4), 1022(ST4), 1023 (ST4), 1024(ST4)[c], 1025(SM3), 1026 (SM3), 1027(SM3), 1028(SM3)[c], 1029(EM3), 1030(EM3), 1031(EM3), 1032(EM3), 1033(EM3)[c].

AUGUST: 1034(HY4), 1035(HY4), 1036(HY4), 1037(HY4), 1038(HY4), 1039(HY4), 1040(HY4), 1041(HY4)[c], 1042(PO4), 1043(PO4), 1044(PO4), 1045(PO4), 1046(PO4)[c], 1047(SA4), 1048 (SA4)[c], 1049(SA4), 1050(SA4), 1051(SA4), 1052(HY4), 1053(SA4).

SEPTEMBER: 1054(ST5), 1055(ST5), 1056(ST5), 1057(ST5), 1058(ST5), 1059(WW4), 1060(WW4), 1061(WW4), 1062(WW4), 1063(WW4), 1064(SU2), 1065(SU2), 1066(SU2)[c], 1067(ST5)[c], 1068 (WW4)[c], 1069(WW4).

OCTOBER: 1070(EM4), 1071(EM4), 1072(EM4), 1073(EM4), 1074(HW3), 1075(HW3), 1076(HW3), 1077(HY5), 1078(SA5), 1079(SM4), 1080(SM4), 1081(SM4), 1082(HY5), 1083(SA5), 1084(SA5), 1085(SA5), 1086(PO5), 1087(SA5), 1088(SA5), 1089(SA5), 1090(HW3), 1091(EM4)[c], 1092 (HY5)[c], 1093(HW3)[c], 1094(PO5)[c], 1095(SM4)[c].

c. Discussion of several papers, grouped by Divisions.